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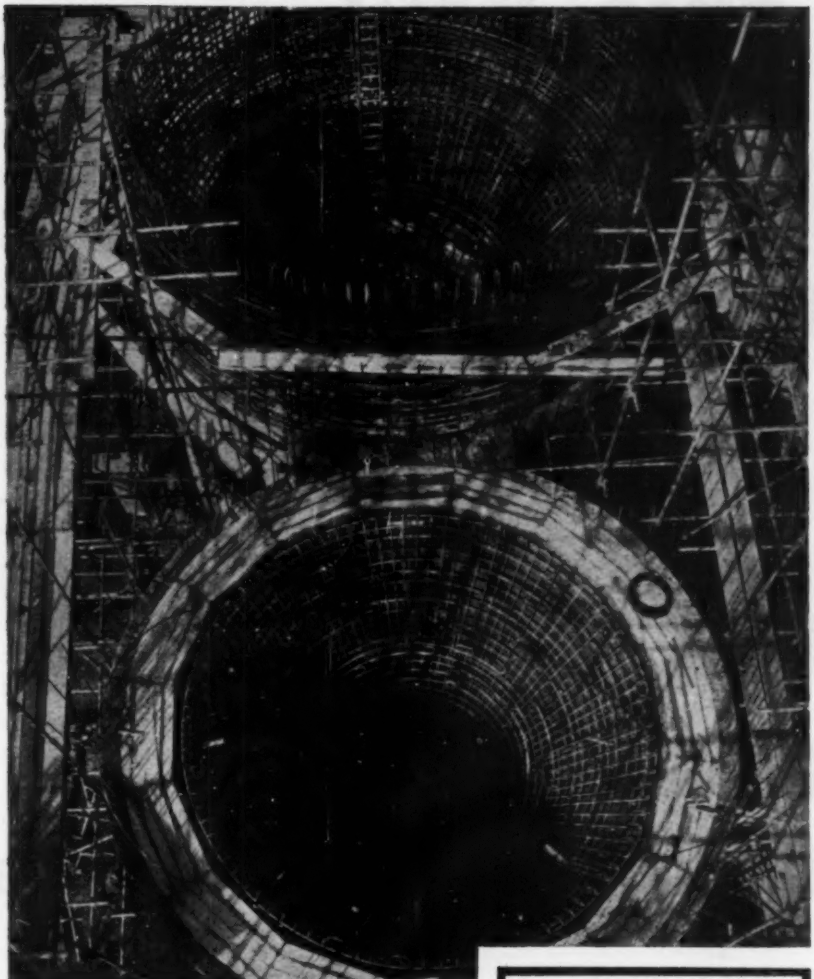

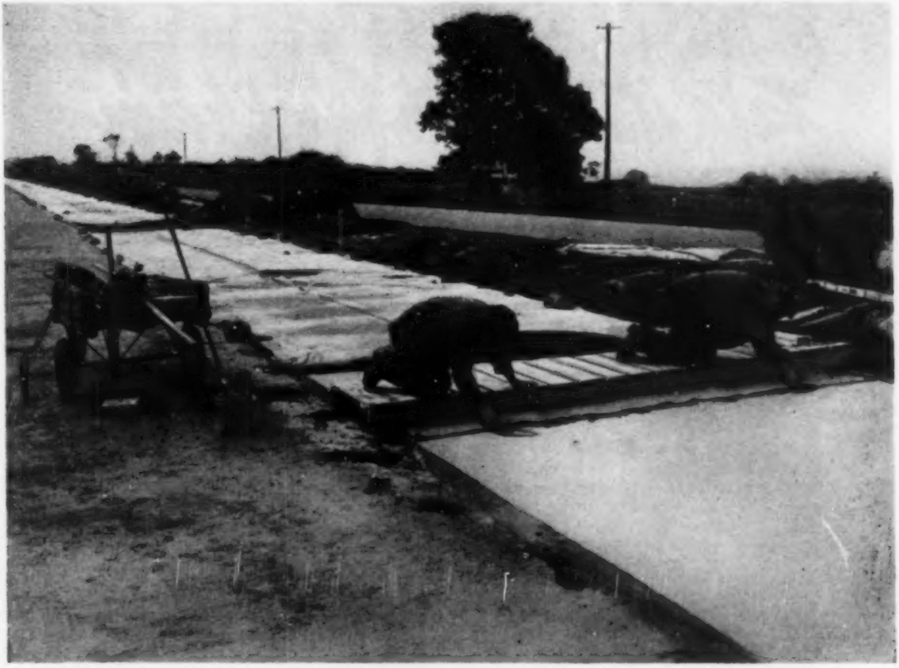

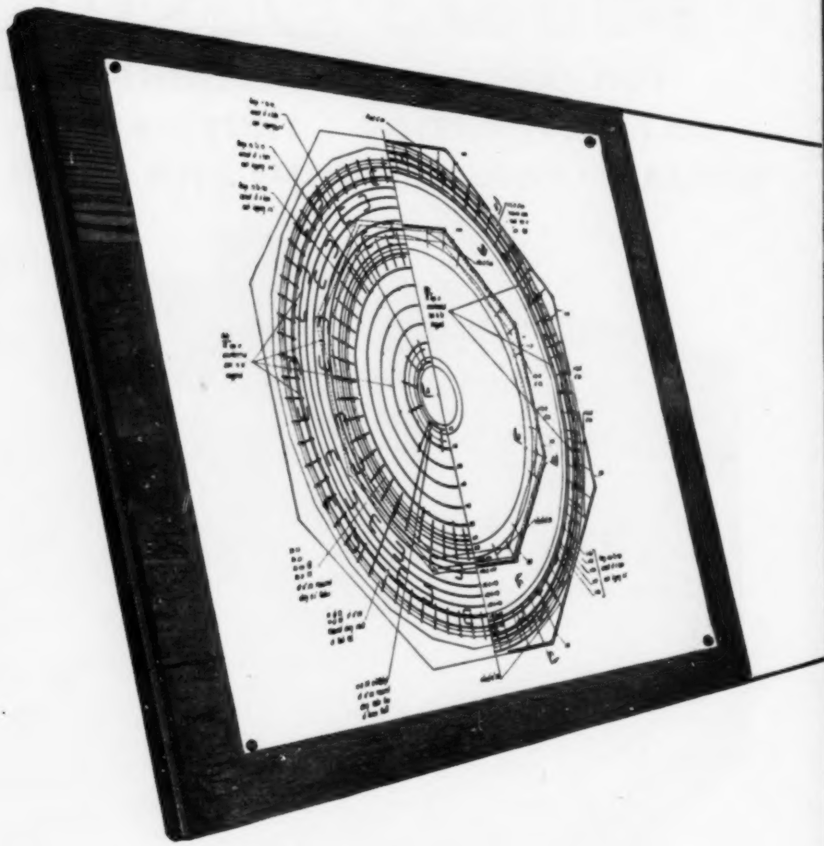

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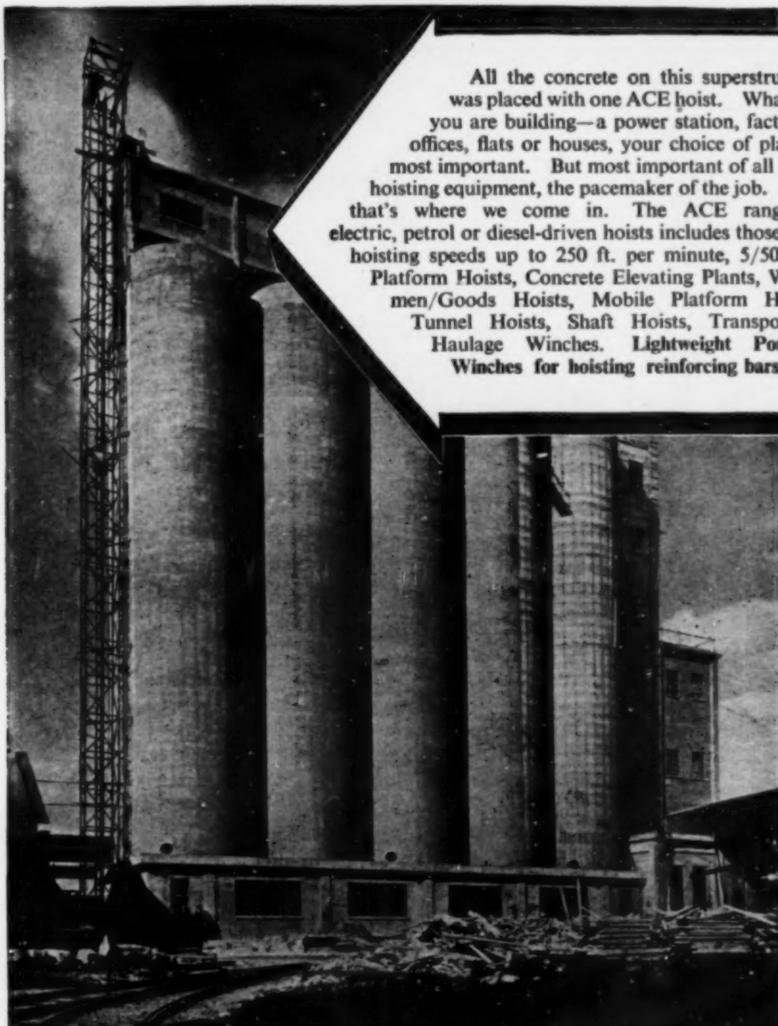

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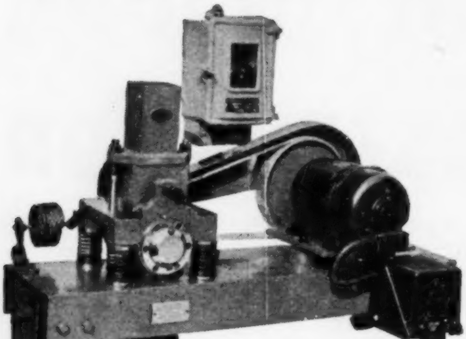

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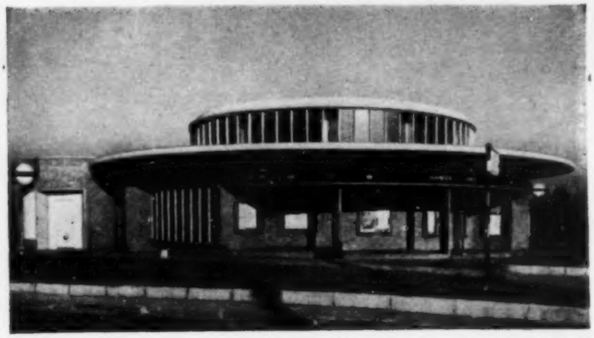

CONCRETE
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Volume XLVI. No. 6. LONDON, JUNE, 1951

EDITORIAL NOTES

Labour in the Construction Industry.

THE helpful attitude now being taken by some trade union leaders in the general prosperity of their industry as apart from the welfare of the workers alone, and a realisation on their part that questions relating to the well-being of the workers cannot be separated from the problems of the employers and building owners, is well shown by the report of the meeting of the Building, Civil Engineering, and Public Works Committee of the International Labour Office held at Geneva in February last. Representatives attended from nineteen countries. The United Kingdom delegation comprised a representative from the Ministry of Labour and another from the Ministry of Works, two representatives of the employers, and two representatives of trades unions ; it is a sign of the changing times in which we live that the two representatives of labour were gentlemen with titles (Sir Luke Fawcett and Sir Richard Coppock) while the representatives of the Government and of the employers were plain Misters.

The recommendations of the committee show that some trades union leaders at any rate are now taking a wide view of their responsibilities. For example, it is recommended that workers in the construction industry should be ready to take any work which they are reasonably capable of doing, to move to areas where work is available, and to be trained in various skills. This conception of the work that may be done by a member of a trade union is directly contrary to the practice in Great Britain to-day. It means that one man would be able and willing to work at more than one trade, which is at present banned by nearly all trades unions. The recommendation is an excellent one, but it is doubtful if it will be readily agreed to by members of craft unions who jealously refuse to allow a worker in one trade to handle the tools of another. The idea is so revolutionary that it seems hardly possible that it can come about even after many years. With the safeguards necessary to prevent men being familiar with several trades but masters of none, with consequent inferior workmanship, it is a suggestion that would greatly reduce the cost of building by removing restrictions that prevent men from working while they wait for another trade to finish. We see no reason, for example, why a man who has become a competent bricklayer could not learn to lay tiles, to drive a lorry, or to attend to a concrete mixer, or why a carpenter employed on shuttering could not be a competent steel fixer, a good musician or a sports champion. The trend towards specialisation must not be allowed to belittle natural ability and aptitudes ; otherwise we class men as machines capable of doing one thing only.

It might also be suggested that a number of the men now classed as labourers be taught a trade. This would also prevent delays, for it is common nowadays for work to be stopped for lack of tradesmen although many labourers are available. It seems that insufficient attention has been paid to the results of the increasing use of mechanical devices on building and civil engineering works. Machinery has done little to reduce the number of craftsmen required on a construction site. It has, however, made large numbers of unskilled labourers redundant, with the result that there is often a surplus of labourers who have no work because no tradesmen are available. It is worth seeing whether some of these labourers who have had experience on construction sites could be trained as craftsmen, for the present trend in the use of machinery in construction is to replace labourers rather than tradesmen. This will increasingly alter our old conceptions of the number of tradesmen compared with labourers required on a construction site, for while one man and a machine may do the work of fifty navvies working with hand tools, the fullest use of machine tools at present available for carpenters, electricians, and others may not save one man in five.

The report contains many suggestions for improving output at a time when it seems that in all parts of the world construction work is being delayed because of the scarcity of materials and of building trades workers. Throughout the report the importance of full employment is emphasised, not only in the building industry but in other industries, and it is realised that the greater the employment in a country as a whole the more work there will be for the construction industry. As is all too common, however, no definition is given of what is meant by full employment. These words should mean a state where everyone is busily and usefully at work. In practice, however, it generally means a state where everyone receives a wage whether or not the work he does is useful or, indeed, whether or not he is doing a full day's work. We do not have to go far to see one man watching another working, with no excuse that the work is so arduous that half the day is needed for rest. This is not full employment. Lack of clear thinking on this subject is shown by a statement in the report that full employment has helped to reduce seasonal unemployment, whereas obviously there can be no seasonal unemployment in a state of full employment.

There are suggestions for improving working conditions such as the provision of shelter when work is interrupted by bad weather, facilities for obtaining food under decent conditions, for washing, for drying clothes, and so on. Much has been achieved in this direction during the past few years, and perhaps it is only their novelty that has caused these amenities to be abused in some cases. But such amenities are among the fundamental rights of human beings. There can be no return to the days when "hands" were wanted and "hands" were discarded, when the hands only were important and the human beings who owned them were left to shift for themselves in matters such as shelter, and sanitary, washing, and clothes-drying arrangements. Such improvements in working conditions are stages in the improvement in living conditions that has been going on, with some setbacks, throughout history. Claims that these amenities cannot be afforded now are no sounder than the arguments of last century which predicted the ruin of the nation if children under twelve years of age were prevented by law from working less than twelve hours a day in a factory or mine. Insofar as higher standards of living increase production the world will be a better place for all.

A Concrete-Lined Tunnel in Scotland.

NOVEL FEATURES ON THE LOCH FANNICH HYDRO-ELECTRIC WORKS.

THE Loch Fannich works is the first stage in the development for hydro-electrical purposes of the basin of the river Conon, Ross-shire, by the North of Scotland Hydro-Electric Board. Water from Loch Fannich flows through the tunnel described in this article to a generating station at Grudie Bridge. The head of water available for the two 12,000-kw. turbines is 530 ft., and will be increased by 20 ft. when a dam is constructed at the loch. The scheme differs from common hydro-electric practice since, instead of placing the intake to the tunnel at the side of the loch and raising the level of the water by a dam, the tunnel is driven under the loch and a connection made about 80 ft. below water level. This method enables much more of the capacity of the loch to be used as a seasonal storage reservoir. The driving and lining of the tunnel and shafts are described in this article by permission of the Board.

The following notes and illustrations on the construction were contributed by Mr. K. McInnes of Messrs. Balfour, Beatty & Co., Ltd., the contractors for the civil engineering works. Technical data and drawings were supplied by the consulting engineers, Sir Alexander Gibb & Partners.

I.—THE TUNNEL.

The length of the tunnel (*Fig.* 2) between Loch Fannich and the portal above the generating station is 19,484 ft. From an adit driven into the hillside at right-angles to the tunnel, the tunnel was driven 4300 ft. to the portal in one direction and 9600 ft. in the opposite direction to meet the heading 4900 ft. long driven from the bottom of the screen-shaft. From the gate-shaft the tunnel was driven under the loch and eventually connected to the loch. The

Fig. 1.—Temporary Concrete Bulkhead No. 1.

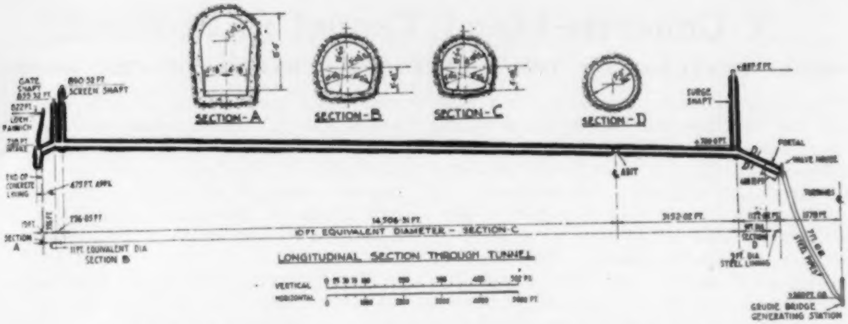


Fig. 2.—Longitudinal and Transverse Sections of Tunnel.

surge-shaft is 3200 ft. from the adit. The tunnel and shafts are lined with concrete. The length of 470 ft. from the gate-shaft to the loch includes 336 ft. of horse-shoe cross section of 11-ft. equivalent diameter ("Section B"); the minimum thickness of the concrete lining is 20 in., and it is strengthened with 8-in. by 5-in. steel arch-ribs at 5 ft. centres as a precaution against shock when the connection to the loch was made. The length of 17,890 ft. from the gate-shaft to the surge-shaft is of horse-shoe cross section of 10-ft. equivalent diameter ("Section C"), and the minimum thickness of concrete over points of rock in the walls is 5 in. and 4 in. in the invert. The length of 1124 ft. from the surge-shaft to the portal is circular, 9 ft. diameter ("Section D"), and is lined with concrete except for a length of 400 ft. from the portal which is lined with a steel pipe backed with concrete. Work commenced on the tunnel in March, 1947, and occupied about 3½ years. The average number of workmen was 350, the greatest number being 700.

Excavation.

The tunnel is generally in hard quartzite (Moine schist), although mica schist was encountered at one part, and no supports were necessary. The total quantity of rock excavated was about 90,000 cu. yd. The long tunnel of 10 ft. equivalent diameter was driven by drilling from jumbos and blasting. The spoil was removed by rocker-shovels loading

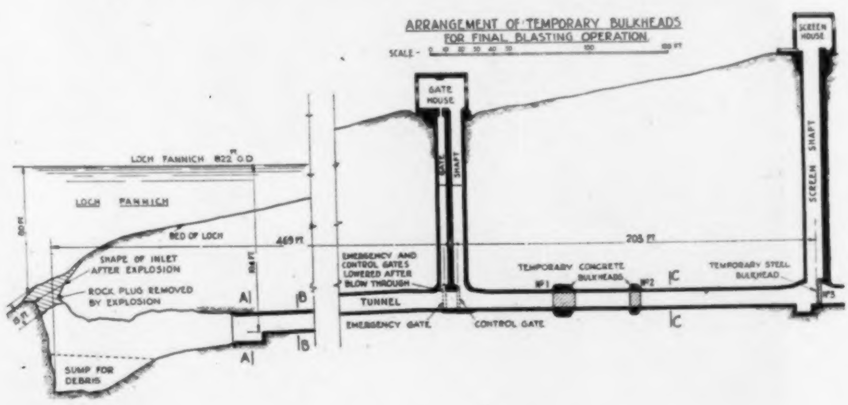


Fig. 3.—Temporary Bulkheads.

2-cu. yd. skips hauled in trains by electric locomotives on 2-ft. gauge track. The speed of driving the tunnel varied considerably with the hardness of the rock, but generally a complete round of drilling, blasting, and removal of spoil took one shift of 8 hours. The greatest advance on one face was nearly 150 ft. in a week with fifteen men in each shift working three shifts a day, or twenty shifts in a week.

Excavation of the 9-ft. diameter circular tunnel was more difficult because of the gradient of about 1 in 23 and the smaller working space. Drifters on column bars were used. The locomotives could not haul more than two loaded skips at a time up the incline.

In the 11-ft. equivalent diameter tunnel, driving was by the heading-and-bench method, using drifters on column bars, to the point where a sump was formed in the floor of the tunnel to contain about twice the estimated amount of rock to be blasted in the break-through to the loch. Beyond this point a small top heading was driven to within 25 ft. of the bed of the loch. The sumps were then excavated, the spoil being removed by a scraper-loader, and a further length of 10 ft. of tunnel forming the throat was excavated upwards towards the loch, thus reducing the cover to 15 ft. As the sump is 18 ft. wide and 48 ft. deep, it was flooded to within a few feet of the roof of the tunnel, and drilling in the roof of the tunnel was done from a pontoon. Pilot holes to ascertain the thickness of the rock were made ahead of the excavation and were later sealed with beechwood plugs and grouted. When only 15 ft. of cover remained, 102 holes terminating about 2 ft. from the bed of the loch were drilled, and 96 of these were loaded with about 1 ton of explosive and all detonated at the same time. The rock at the intake is a quartz-mica schist (granulite) and the amount blasted by the explosion was about 270 tons. To reduce the effect of an inrush of water, the tunnel from the gate-shaft towards the loch was filled with water until the water in the shaft was about 12 ft. below the level of the water in the loch. The back pressure of this water also helped the shattered rock to fall into the sump. To ensure that the water would not rush through the tunnel, temporary bulkheads were constructed (*Fig.* 3). Instruments at the foot of the gate-shaft recorded the

Fig. 4.—Travelling Shuttering for Walls of 10-ft. Tunnel.

force of the explosion, which it was estimated would be about 120 tons per square foot. Bulkhead No. 1, nearest the loch, was a concrete plug 12 ft. 6 in. thick. Bulkhead No. 2 was 36 ft. from the first bulkhead and was a concrete plug 6 ft. 6 in. thick. If the shock pressure of nearly 10,000 tons on bulkhead No. 1 caused this bulkhead to fracture, then bulkhead No. 2 would have to resist only the pressure of the water without the was again available down the gate-shaft. This is only the second time that this method of connecting a tunnel to a lake has been used in this country. After the explosion, the two permanent steel gates were lowered to the bottom of the gate-shaft so that the water between the gate-shaft and bulkhead No. 1 could be removed and the concrete bulkheads demolished. The removal of the bulkheads took a week. Vertical and hori-

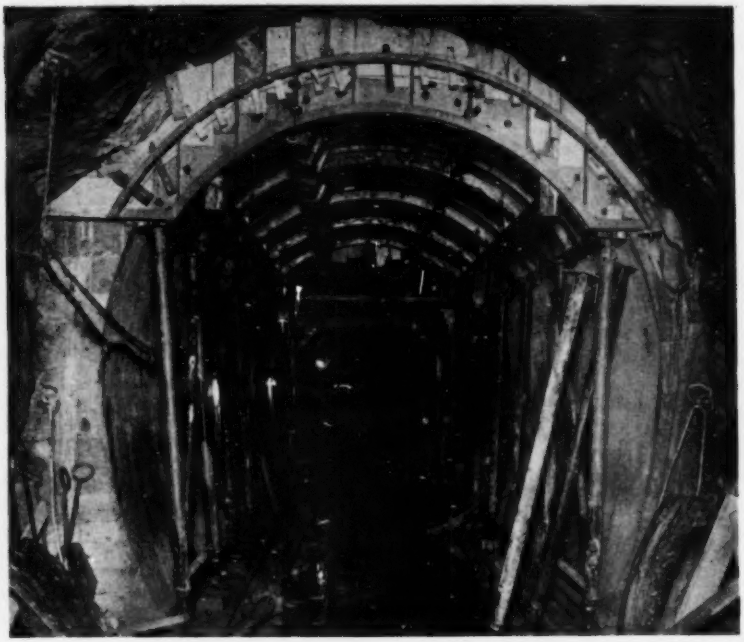


Fig. 5.—Travelling Shuttering for Roof of 10-ft. Tunnel.

shock of the explosion. Bulkhead No. 3 was a temporary gate of steel joists at the bottom of the screen-shaft, and was an additional precaution. Bulkhead No. 1, however, resisted satisfactorily the full force of the explosion. While the gate-shaft was temporarily closed for lining, it was necessary to leave openings in the concrete bulkheads for the passage of skips of spoil from the incomplete tunnel under the loch. The openings were closed during blasting operations by 12-in. gates of timber baulks (*Fig.* 1), and were filled with concrete when access

zontal cuts were made by a machine, and the remainder of the concrete was demolished by the plug-and-feather method.

The gate-shaft and screen-shaft were sunk from the surface, drilling being done by jack-hammers and sinkers and the spoil being loaded by hand into crane buckets. The surge-shaft was excavated partly upwards from the tunnel. A small vertical shaft, or stope, was driven from the tunnel to the surface, and was then enlarged to the full size by excavating from the surface, the spoil being thrown

down the stope, at the bottom of which was a steel hopper outlet through which the rock was discharged into skips in the tunnel.

Concrete and Concrete Plant.

There are about 45,000 cu. yd. of concrete in the lining and elsewhere in the tunnel and shafts. The cement was brought in 1-cwt. paper bags by boat to Invergordon and thence by road to the site, where it was stored in cement sheds at the adit and intake. Sand was delivered by road from a pit at Beauly. Much of the rock from the tunnel was suitable for aggregate, and a crushing, washing, and screening plant was installed at the adit to produce aggregates, conforming to British Standard No. 882, in three sizes, namely 1½ in. to ¾ in., ¾ in. to ⅜ in., and ⅜ in. to $\frac{3}{16}$ in., which were mixed to give the required grading.

The mixture of the concrete was designated by the number of hundredweights of cement to be mixed with 12 cu. ft. of sand and 20 cu. yd. of aggregate (or slight variations of these quantities) and the maximum size of the aggregate. For example, concrete containing 3 cwt. of cement and 1½-in. aggregate was called 3/1½. The concrete in the lining of the tunnel of 10-ft. equivalent diameter is 5/1½. The maximum size of the aggregate in concrete placed by pump is ¾ in., the concrete in the gate-shaft, screen-shaft, and lining the tunnel of 11-ft. equivalent diameter being 6/¾. Tests were made regularly of the materials, and slump tests and cubes were made of nearly every placing of concrete. Most of the concrete was weigh-batched at plants at the adit and intake.

The batching plant at the adit had four compartments which were filled with aggregate from the stockpiles by a derrick grab-crane. The weigh-hopper was of the beam-balance type with four sets of levers and weights, and discharged into the chute-hopper of a 1-cu. yd. mixer. The bags of cement for a batch of concrete were broken into a skip which was on rails running the length of the cement shed and out to the batcher. The skip was emptied into the hopper of a hoist, which discharged the cement into the chute-hopper of the mixer. The concrete was discharged from the mixers into skips on tracks in front of the mixer and conveyed into the adit.

The batching plant at the intake also comprised four compartments, and was

Fig. 6.—Timber Shutter for Upper Part of Walls of 11-ft. Tunnel.

fed by skips on tracks over the bins. The weigh-batcher was of the turntable type and could feed into the hopper of either of two 21/14 mixers. Bags of cement were slid down a chute from the cement shed and broken into the hoppers. The mixers discharged into skips which were hauled by a winch up a ramp to lifts in the screen-shaft by which they were taken down into the tunnel.

Lining the Tunnels.

The lining of the tunnel of 10-ft. equivalent diameter was done in four stages. The haunches were concreted first, and rails were spiked to wooden plugs driven into holes drilled in the haunches. The rails carried the steel carriage of the travelling shutters for the walls and roof. The shutter for the walls (*Fig.* 4) comprised the carriage, which allowed traffic to pass through the centre of the tunnel, and steel shutters mounted on both sides of the carriage. Jacks at the top and bottom enabled the shutters to be moved in or out for striking or resetting. The length of the shutter was generally 70 ft., but one length of 100 ft. was also used. The plates had ports about halfway up and at 5 ft. centres horizontally. The concrete was brought into the tunnel in ½-cu. yd. trays mounted on bogies, and was shovelled through the ports until the level of the concrete reached the bottom of the ports, which were then closed and the concrete shovelled over the top of the shutter. Steel shutters supported on props with screw-jacks (*Fig.* 5) were used for the roof. For moving the shutters, another

Fig. 7.—Lining of 11-ft. Tunnel complete except for Invert.

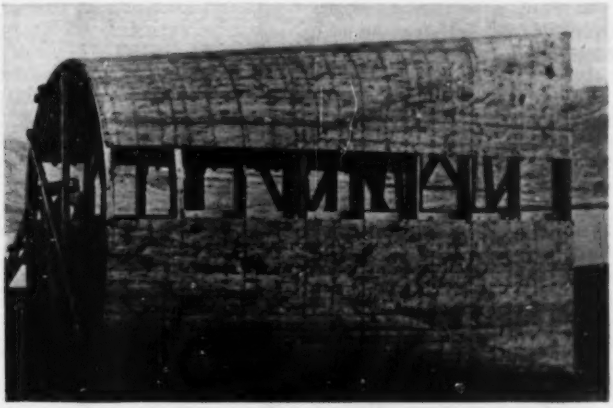


Fig. 8.—Transition Shutter at Gate-shaft: From 11-ft. by 10-ft. Rectangle to 10-ft. Horseshoe Cross Section.

carriage was run under the shutter which was then lowered on to it. The shutter had doors covering the crown of the arched roof. Concrete was also brought in on ½-cu. yd. trays, which were also used as platforms, and one team of men commenced at one end of the shutter and filled up to the level of the door. Another team followed, closing the doors in turn and filling over the top of each. Since only two men could do the last operation, the roof-shutters were only 20 ft. and 40 ft. in length, but there were more roof shutters than wall shutters. For concreting the invert, steel beams were placed at 6 ft. centres across the tunnel to support the track clear of the invert, which was then trimmed and concreted. A shaped screed sliding on concrete nibs formed on the walls was used.

In the steel-lined part of the 9-ft. diameter circular tunnel, a concrete invert was laid to carry the track, on which 16-ft. sections of the steel lining were rolled into place. The sections were then welded together, and generally

Fig. 9.—Inner View of Continuously-moving Shutter for Surge-Shaft.

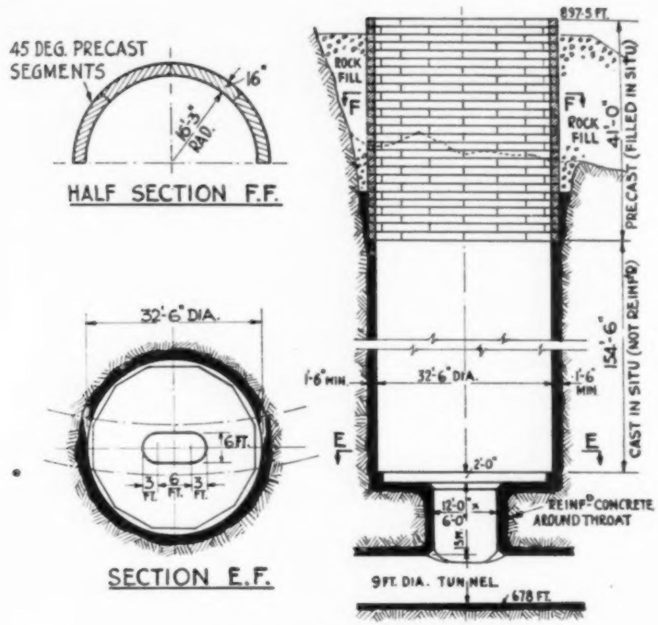


Fig. 10.—Surge-Shaft.

a 16-ft. length was concreted at a time. A 4-in. concrete pump and a 14/10 mixer were installed in a sump in the tunnel. Batches of aggregate were brought in from the adit batching plant and stored with the cement on a platform behind the mixer. The pipes from the pump were run over the steel lining. To allow access to assemble the pipes and ram the concrete, extra excavation was taken out of the roof.

Fig. 11.—Outer View of Continuously-moving Shutter for Surge-Shaft.

The first stage of lining the concrete part of the circular tunnel was to concrete the invert by means of the concrete pump. The timber shutter, which was used for the sides and roof was in halves, the joint being at the centre of the crown. Upon removal of a key-piece, the halves could be pulled together for striking, and the shutter was then rolled forward on small timber rollers and wedged apart for resetting. The concrete was placed behind the shutter by the concrete pump.

Access to line the tunnel of 11-ft. equivalent diameter was generally only possible down the gate-shaft, one well of which was occupied by the skip for removing spoil and the other well by ladders, pipes, and the temporary gate. Therefore a 4-in. concrete pump and a 21/14 mixer were installed at the top of the shaft. Batches of cement and aggregate were mixed dry in one of the batching-plant mixers and carried by belt-conveyors to the hopper of the mixer at the shaft. Concrete was pumped down the 125-ft. shaft successfully without choking by providing a vent at the top of the vertical pipe and an easy bend at the

bottom. A timber shutter 80 ft. long was provided (*Fig.* 6), a length of 40 ft. being filled at a time. The concrete was brought up to about 1 ft. over the crown of the arch, and after 24 hours was filled to the rock. This method avoided undue weight being imposed on the shutter since the concrete, which is reinforced by steel ribs as seen in *Fig.* 6, is several feet thick where an excessive amount of rock has been excavated. *Fig.* 7 is a view in this part of the tunnel before the invert was concreted. Closure lengths of the lining near the bottom of the screen-shaft were also concreted by pump, and at one time the pumping distance was about 260 ft. horizontally from the pump at the gate-shaft to the top of the screen-shaft, 160 ft. down the screen-shaft, and 360 ft. along the tunnel. Difficulties experienced at first were overcome by creating a vacuum in the vertical pipe by exhausting air from the top vent. A small section of this tunnel was lined with concrete shot into place.

The special shuttering required for the bends, transitions, and connections with the shafts was of timber, although one easy bend in the 10-ft. tunnel was shuttered by a 20-ft. length of the straight steel shutters. The shutters for transition sections were generally made by attaching narrow boards to shaped centres, and were erected completely above ground, cut into sections, and taken down into the tunnel. The transition shutter in *Fig.* 8 is for a part of the tunnel where the cross section changes from a rectangle 11 ft. by 10 ft. to a 10-ft. horseshoe section.

II.—THE SHAFTS.

The surge-shaft (*Fig.* 10) is 32 ft. 6 in. diameter and about 200 ft. deep, and has a throat 6 ft. wide, 12 ft. long, and 13 ft. deep connecting to the tunnel. It was intended to use a steel shutter in which concrete could be placed to a depth of 6 ft. at a time, but, to accelerate the work, this shutter was strengthened, and converted to a continuously-moving form (*Figs.* 9 and 11). Fourteen jack-rods (*Figs.* 12 and

Fig. 12.—Jack for Continuously-moving Shutter for Surge-Shaft.

Fig. 13.—Jack Rods in Surge-Shaft.

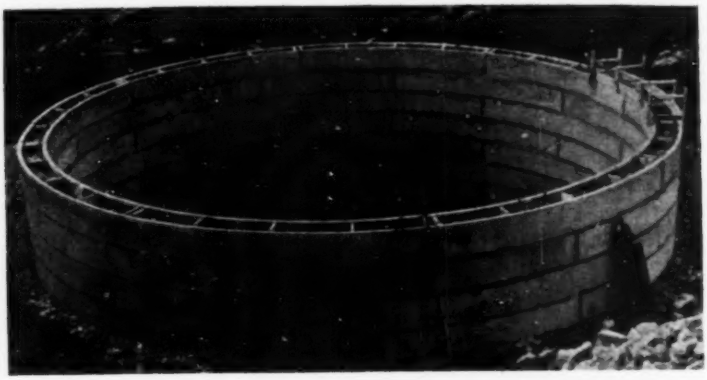


Fig. 14.—Precast Segmental Wall at Top of Surge-Shaft.

13) were fixed to the sides of the shaft. The concrete was mixed at the adit batching plant and brought through the tunnel in skips, which were hoisted through the throat to the shutter by a 10-tons derrick crane at the top of the shaft. After movement of the shutter had started, a smaller crane was installed on the ledge around the throat to lift the skips through the throat, the derrick crane then lifting them from the ledge to the shutter, thereby saving time. A depth of about 3 ft. of concrete was kept in the shutter, and it moved upwards about 6 in. per hour. The upper part of the surge-shaft (*Fig.* 14) is constructed of hollow precast concrete segments in courses each 18 in. deep. There are eight 45-deg. segments in each ring, and after they had been bedded in 1 : 2 mortar they were filled with $5/\frac{3}{4}$ concrete reinforced with ring bars set in slots in the tops of the webs.

The gate-shaft is 125 ft. deep and the timber shutters for the two wells (*Fig.* 15) were each 5 ft. 6 in. deep; enough

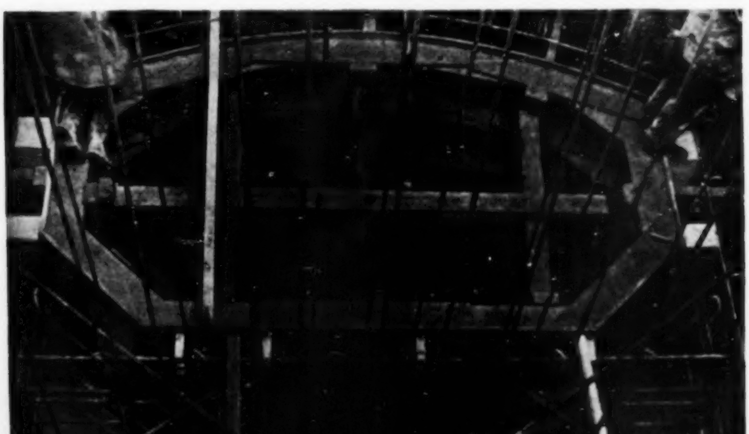


Fig. 15.—Shutter for One Well of Gate-Shaft.

shutters were made to concrete one-third of each well. The materials were measured in batch boxes and mixed in a 21/14 mixer at the top of the shaft which discharged into a skip handled by a derrick crane. The average time taken to set the shutter, fix the reinforcement, and concrete a lift was about 36 hours, but varied considerably according to the distance down the shaft at which concrete was being placed.

The screen-shaft is 160 ft. deep and lifts were installed in it for taking men and materials down to the tunnel; while the shaft was being lined the lifts were removed, and a 10-in. pipe with a hopper at the upper end and a sector-gate at the lower end were installed in the shaft for passing concrete into the tunnel. Later it was possible to finish lining the main tunnel from the adit. Timber shutters each 5 ft. 6 in. deep were used for the screen-shaft, sufficient shutters being made to line nearly half the shaft. As lining the 11-ft. tunnel was finished by this time, the pipe from the concrete pump at the gate-shaft was laid to the top of the screen-shaft and down to the shutter. It was possible to fix the reinforcement well ahead of the lining, and a shutter could be set and concreted in 24 hours.

Pressure of Concrete on Shuttering.

The probable pressures on vertical shuttering in *Table* I are based on data derived from tests and given by M. Guerrin in a recent number of "La Technique de Travaux". It is seen that even if the extreme cases of liquid and earth-damp concretes are ignored, the pressures vary by 75 per cent. to 150 per cent. depending upon the consistency. If the rate of placing is increased from about ⅓ cu. yd. per hour to about 1⅓ cu. yd. per hour, the pressures increase up to 25 per cent. and this is especially so if the concrete is in wide walls or beams. If the concrete is in thin walls or beams or is vibrated, the rate of placing appears to be of little importance. An increase in temperature causes a decrease in pressure; for example, the pressure may decrease from 25 per cent. to 40 per cent. if the temperature rises from 60 deg. to 100 deg. F. The "silo" effect in narrow shuttering may result in a decrease up to 30 per cent. of the pressure of ordinary concrete, and a decrease up to 10 per cent. with vibrated concrete. The "silo" effect does not occur with liquid concrete, but in the case of "earth damp" concrete it may reduce the pressure by a half. Tests in the U.S.A. have shown that with liquid concrete the pressure may be about 50 per cent. greater for a rich mixture than for a lean mixture. The influence of the nature of the shuttering is negligible, although for ordinary concrete the pressure on steel shuttering may be about 3 per cent. greater than on wooden shuttering.

TABLE I.—MAXIMUM PRESSURE (lb. per square foot) OF WET CONCRETE ON SHUTTERING.

	Mass of indefinite width		Columns (about 18 in. square)		Walls (about 5 in. thick)	
Rate of placing (cu. ft. per hour)	10	35	10	35	10	35
Consistency:						
"Earth damp"	332	415	167	215	84	87
Suitable for vibration	485	523	504	528	457	463
Ordinary	733	922	582	704	659	659
Liquid	1230	1530	1230	1530	1230	1530

Book Reviews.

"Reinforced Concrete Designer's Handbook." By C. E. Reynolds. Fourth Edition (revised). (London: Concrete Publications Ltd. 1951. Price 18s.)

In this edition the opportunity has been taken to include additional tables and notes giving the requirements of the British Standard Code, CP.114 (1948), for working stresses, bond, slabs spanning in two directions, the moment of resistance of beams and slabs, and the safe loads on columns. The requirements of the D.S.I.R. Code, the London By-laws (1938), and the code of the Institution of Civil Engineers for Liquid-containing Structures are retained. New tables and examples give an improved method of calculating the bending moments on rectangular slabs subjected to triangularly-distributed pressure, and the loads on groups of vertical and inclined piles in wharves and jetties. Other new matter includes a calculation chart for fixed-end symmetrical arches, the general formulæ for the longitudinal forces in prismatic slab structures, and formulæ for I-beams. There are 75 tables and about 265 pages of explanatory text, formulæ, diagrams, examples, and specifications.

"Modern Bridge Construction." By F. Johnstone Taylor. Second Edition. (London: The Technical Press, Ltd. 1951. Price 30s.)

This book of about 300 pages deals mainly with steel bridges. The design of reinforced concrete bridges of beam, fixed-arch, and rigid-frame types is dealt with in 18 pages, and their construction in 39 pages. The formulæ and design data are not sufficiently explained to be of great value, and some statements are misleading; for example, that the tensile resistance of the concrete is sometimes considered in the design of arches. Also, Waterloo bridge is not an arch structure, although the girder may appear to be arches because the soffit is curved. In some places the book has not been brought up-to-date, since Berwick bridge can hardly be described as "recently completed", and the Traneberg bridge of 563-ft. span is not the longest reinforced concrete arch, as the Sandö bridge, completed in 1943, has a span of 866 ft.

"Der Stahlbeton in Beispielen. No. 2. Durchlaufende Platten." By Adolf Kleinlogel. (Berlin: Wilhelm Ernst & Sohn. 1951. Price 9 D.M.)

In this second of a series of small books the author deals with the calculation of continuous slabs in six numerical examples illustrating different arrangements of spans and combinations of loading, slabs spanning in two directions, and the application of influence lines to rolling loads. The examples are taken from actual work, and are fully worked out. The main purpose of these booklets is to provide students with ready answers to points not covered in an incomplete technical education.

Publications Received.

"Sources of Road Aggregate in Great Britain." Second Edition. (London: H.M. Stationery Office. 1951. Price 3s.)

"Symposium on Use of Pozzolanic Materials in Mortars and Concretes." 208 pages. (Philadelphia: American Society for Testing Materials. 1950. Price 2.50 dollars.)

International Association for Bridge and Structural Engineering.

Papers printed in the tenth volume of the Publications of the International Association for Bridge and Structural Engineering include the following. A report of the test of the 160-ft. prestressed concrete girder for the Walnut Lane bridge, Philadelphia, by M. Fornerod; A theory and method of design of the girders of a bridge with several longitudinal girders, taking into account the torsional resistance of the girders, by C. Massonett; The measurement of the stresses in a bent reinforced concrete beam with special reference to the bond of the bars, by L. P. Brice; An analysis of continuous beams on elastic foundations and deformed by shearing forces only, by A. Holmberg; A simplified method of analysing Vierendeel girders, by F. Stüssi; The best manner in which the material in a structure can be used, by R. Vallette; Test of a plastic model of a curved reinforced concrete viaduct, by G. Wästlund and L. Östlund. The Publications (no price stated) are obtainable from Verlag Leemann, Zurich.

Beams with Variable Moment of Inertia.—I.

By B. ERIKSEN, A.M.I.Struct.E.

In the following is described briefly a method of calculating, by means of fixed points, the bending moments (and thence the shearing forces) in continuous or other restrained beams in which the moment of inertia varies throughout the span. This is an adaptation of the formulæ and other data for beams with constant moment of inertia given by the writer in this journal for January, February, and March, 1948.

The principal symbols are given in *Fig.* 1 (symmetrical beams) and *Fig.* 3 (unsymmetrical beams).

Symmetrical Beams.

It is assumed that the moment of inertia varies according to

$$n_x = 1 - (1 - n)\phi^{2r} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which $\phi = \frac{2x}{l}$ and $2r$ are indices depending on the shape. The different shapes of the soffit of beams with three values of $2r$ are shown in *Fig.* 2. If the cross section of the beam is rectangular,

$$d_x = \frac{d_c}{\sqrt[3]{1 - (1 - n)\phi^{2r}}} \quad . \quad . \quad . \quad . \quad (2)$$

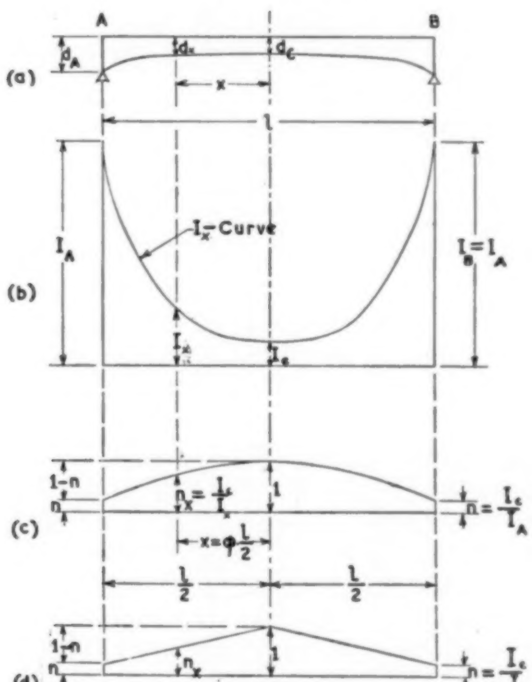
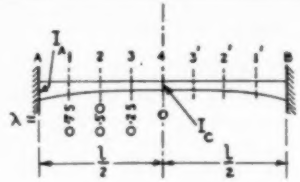
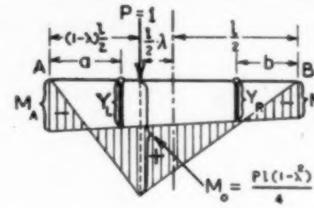


Fig. 1.

TABLE No. 1.





2r	$n=\frac{I_c}{I_A}$	Values of $-\frac{Y_L}{a}$ for P = 1 — Load positions 1	2	3	4	Values of $-\frac{Y_R}{b}$ for P = 1 — Load positions 1	2	3	4
1	1·0	0·2051	0·3281	0·3809	0·3750	0·1230	0·2344	0·3223	0·3750
	0·4	0·1877	0·3175	0·3843	0·3871	0·1238	0·2389	0·3328	0·3871
	0·3	0·1838	0·3152	0·3850	0·3898	0·1240	0·2399	0·3348	0·3898
	0·2	0·1794	0·3125	0·3859	0·3928	0·1242	0·2411	0·3373	0·3928
	0·15	0·1771	0·3111	0·3863	0·3945	0·1243	0·2417	0·3387	0·3945
	0·1	0·1746	0·3096	0·3868	0·3962	0·1244	0·2423	0·3402	0·3962
	0·05	0·1719	0·3079	0·3873	0·3981	0·1245	0·2430	0·3417	0·3981
2	1·0	0·2051	0·3281	0·3809	0·3750	0·1230	0·2344	0·3223	0·3750
	0·4	0·1908	0·3209	0·3840	0·3835	0·1238	0·2384	0·3301	0·3835
	0·3	0·1880	0·3195	0·3846	0·3852	0·1239	0·2391	0·3317	0·3852
	0·2	0·1851	0·3181	0·3852	0·3869	0·1241	0·2399	0·3333	0·3869
	0·15	0·1835	0·3173	0·3856	0·3878	0·1242	0·2404	0·3341	0·3878
	0·1	0·1821	0·3166	0·3859	0·3887	0·1243	0·2408	0·3349	0·3887
	0·05	0·1805	0·3158	0·3862	0·3897	0·1244	0·2412	0·3358	0·3897
3	1·0	0·2051	0·3281	0·3809	0·3750	0·1230	0·2344	0·3223	0·3750
	0·4	0·1939	0·3233	0·3834	0·3811	0·1237	0·2376	0·3281	0·3811
	0·3	0·1918	0·3224	0·3840	0·3822	0·1239	0·2383	0·3292	0·3822
	0·2	0·1897	0·3215	0·3843	0·3833	0·1240	0·2389	0·3303	0·3833
	0·15	0·1887	0·3211	0·3846	0·3839	0·1241	0·2392	0·3309	0·3839
	0·1	0·1876	0·3206	0·3848	0·3845	0·1241	0·2395	0·3315	0·3845
	0·05	0·1865	0·3201	0·3851	0·3851	0·1242	0·2398	0·3321	0·3851
4	1·0	0·2051	0·3281	0·3809	0·3750	0·1230	0·2344	0·3223	0·3750
	0·4	0·1962	0·3248	0·3829	0·3795	0·1237	0·2371	0·3268	0·3795
	0·3	0·1947	0·3242	0·3832	0·3803	0·1238	0·2376	0·3276	0·3803
	0·2	0·1931	0·3236	0·3836	0·3811	0·1239	0·2380	0·3284	0·3811
	0·15	0·1922	0·3233	0·3838	0·3816	0·1240	0·2383	0·3288	0·3816
	0·1	0·1914	0·3229	0·3840	0·3820	0·1240	0·2385	0·3292	0·3820
	0·05	0·1906	0·3227	0·3842	0·3824	0·1241	0·2388	0·3296	0·3824

If unit moment is applied at one end of the beam the slope at that end is given by

$$\alpha = \frac{l}{3EI_e}\left[1 - \frac{3(1-n)(r+1)}{(2r+1)(2r+3)}\right] \quad . \quad . \quad . \quad . \quad (3)$$

The slope at the other end to that at which the moment is applied is given by

$$\beta = \frac{l}{6EI_c}\left[1 - \frac{3(1-n)}{(2r+1)(2r+3)}\right] \quad . \quad . \quad . \quad . \quad (4)$$

When the slopes α and β are known, the positions a and b of the fixed points can be determined. In the application of the fixed-point method, the ordinates Y_L and Y_R to the closing line of the bending-moment diagram at the left-hand and right-hand fixed points are required. In *Table* No. 1 are given coefficients of $-\frac{Y_L}{a}$ and $-\frac{Y_R}{b}$ for unit load acting at any one of four positions on one half of a symmetrical span, and for values of $2r$ equal to 1, 2, 3, and 4, and for most common values of n. The coefficients, which are calculated by a complex formula based on a rigid analysis of the slopes, enable the bending moments at, and the influence lines for, each of the positions to be readily calculated.

Influence lines derived from the coefficients enable the effect of most types of loads to be determined, but for a uniformly-distributed load it can be shown by direct analysis that $Y_L = -\frac{awl}{4}$ and $Y_R = -\frac{bwl}{4}$.

An example of the design of a structure with symmetrical beams having variable moment of inertia will be given in the second part of this article.

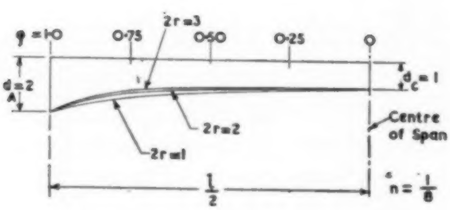


Fig. 2.

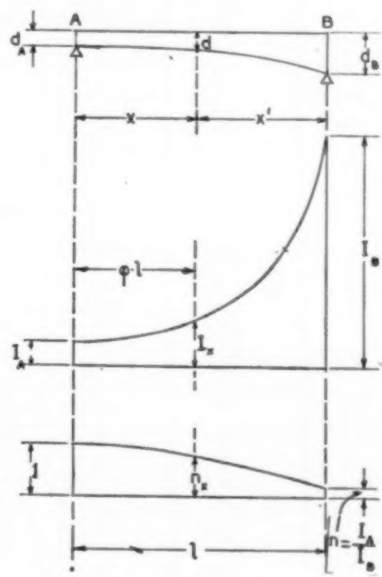
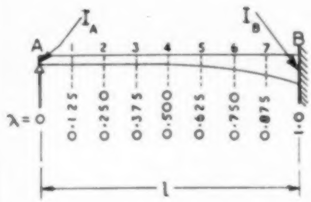
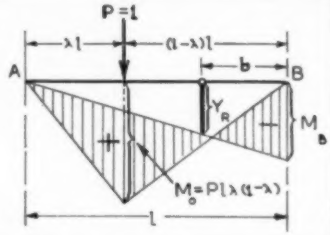


Fig. 3.

TABLE NO. 2.



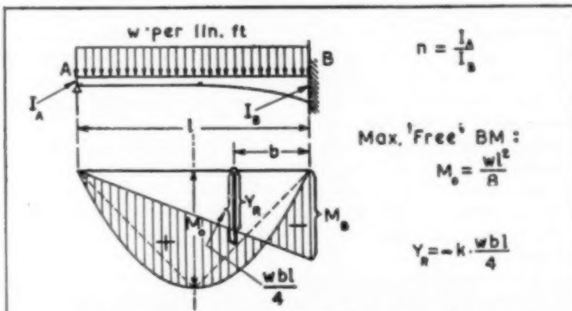


2r	$n=\frac{I_A}{I_B}$	Values of $-\frac{Y_B}{b}$ for P=1 — Load positions 1	2	3	4	5	6	7
1	1·0	0·1230	0·2344	0·3223	0·3750	0·3809	0·3281	0·2051
	0·5	0·1225	0·2305	0·3113	0·3542	0·3504	0·2930	0·1772
	0·4	0·1223	0·2293	0·3082	0·3482	0·3417	0·2829	0·1692
	0·3	0·1221	0·2280	0·3045	0·3414	0·3315	0·2713	0·1600
	0·2	0·1220	0·2266	0·3003	0·3333	0·3198	0·2578	0·1493
	0·1	0·1217	0·2248	0·2942	0·3239	0·3059	0·2418	0·1366
	0·05	0·1215	0·2238	0·2924	0·3185	0·2980	0·2360	0·1293
2	1·0	0·1230	0·2344	0·3223	0·3750	0·3809	0·3281	0·2051
	0·5	0·1227	0·2318	0·3143	0·3585	0·3548	0·2956	0·1774
	0·4	0·1226	0·2312	0·3123	0·3544	0·3482	0·2876	0·1706
	0·3	0·1225	0·2305	0·3102	0·3501	0·3413	0·2790	0·1634
	0·2	0·1224	0·2301	0·3079	0·3454	0·3339	0·2699	0·1555
	0·1	0·1223	0·2290	0·3055	0·3403	0·3259	0·2599	0·1470
	0·05	0·1223	0·2285	0·3042	0·3376	0·3216	0·2546	0·1425
3	1·0	0·1230	0·2344	0·3223	0·3750	0·3809	0·3281	0·2051
	0·5	0·1228	0·2328	0·3167	0·3628	0·3604	0·3010	0·1800
	0·4	0·1228	0·2323	0·3155	0·3600	0·3557	0·2949	0·1752
	0·3	0·1227	0·2319	0·3141	0·3572	0·3508	0·2884	0·1691
	0·2	0·1227	0·2314	0·3128	0·3542	0·3458	0·2817	0·1630
	0·1	0·1226	0·2310	0·3113	0·3510	0·3404	0·2746	0·1566
	0·05	0·1226	0·2308	0·3106	0·3493	0·3376	0·2709	0·1532
4	1·0	0·1230	0·2344	0·3223	0·3750	0·3809	0·3281	0·2051
	0·5	0·1229	0·2332	0·3183	0·3660	0·3650	0·3059	0·1838
	0·4	0·1229	0·2329	0·3174	0·3640	0·3615	0·3011	0·1791
	0·3	0·1228	0·2326	0·3165	0·3620	0·3578	0·2961	0·1743
	0·2	0·1228	0·2324	0·3156	0·3598	0·3542	0·2909	0·1693
	0·1	0·1228	0·2321	0·3146	0·3577	0·3503	0·2856	0·1642
	0·05	0·1227	0·2319	0·3141	0·3566	0·3484	0·2830	0·1616

Unsymmetrical Beams.

When one end of a beam or other member is freely supported it is often more economical if the depth (or width) increases from a minimum at the free end to a maximum at the support over which the beam is continuous. This shape is fairly common for columns of framed structures as well as for end spans

TABLE NO. 3.



Values of k for varying n and 2r

2r	Values of n: 1·0	0·5	0·4	0·3	0·2	0·1	0·05
1·00	1·0	0·9333	0·9143	0·8923	0·8666	0·8363	0·8191
2·00	1·0	0·9412	0·9268	0·9114	0·8947	0·8767	0·8671
3·00	1·0	0·9524	0·9416	0·9302	0·9184	0·9059	0·8995
4·00	1·0	0·9615	0·9531	0·9444	0·9355	0·9262	0·9215

of girder bridges. The adaptation to this case of the data for the symmetrical beams is useful. *Fig.* 3 shows a beam of this shape, and on the same diagram are given the curves of I_x and n_x. With the symbols in the diagram the slope at B due to unit bending moment applied at B is given by

$$\alpha_B = \frac{l}{3EI_A}\left[1 - \frac{3(1-n)}{2r+3}\right] \quad . \quad . \quad . \quad . \quad (5)$$

The slope at A due to unit bending moment at B is given by

$$\beta = \frac{l}{6EI_A}\left[1 - \frac{6(1-n)}{(2r+2)(2r+3)}\right] \quad . \quad . \quad . \quad (6)$$

The coefficients of $-\frac{Y_L}{a}$, when calculated on a similar basis to that for symmetrical beams, are given in *Table* No. 2 for unit load acting at any one of eight positions on the beam, and enable bending-moment diagrams and influence lines to be drawn as before.

Table No. 3 applies to a uniformly-distributed load on an unsymmetrical beam. The ordinates at the fixed points in this case can be expressed as k times the ordinate for a symmetrical beam, that is, $Y_R = -\frac{bwl}{4}k$. Values of k are given in *Table* No. 3.

(*To be concluded.*)

Continuously-Reinforced Concrete Roads.

AT the last annual meeting of the United States Highway Research Board, reports on three continuously-reinforced concrete roads were given, the following details of which are from "Engineering News-Record".

A test road one mile long laid in California in the summer of 1949 has an 8-in. slab in two 12-ft. strips. Half the length is reinforced with ½-in. low-strength bars at 4 in. centres, and the other half with high-strength bars at 5 in. centres. The bars are continuous through the construction joints, and amount to 0·62 per cent. and 0·50 per cent. respectively of the cross-sectional area. Attached to the bars are 184 strain gauges and 72 temperature compensation gauges. Only about 5 per cent. of the strain gauges were effective at the end of 18 months. The average stress in bars across cracks after seven months was about 50,000 lb. per square inch in low-strength bars and 81,000 lb. per square inch in high-strength bars. In both cases the maximum stress was reached in less than twelve months. At the end of the first year the average number of cracks in 100 ft. of road was generally 22. There were fewer cracks in the older concrete on one side of each construction joint and more in the first section laid each day. As the reinforcement is continuous, it is probable that the greater cracking is due to the fact that the newer concrete has less strength to resist the shrinkage pull across the joint.

A road laid in Illinois in 1947–48 is 5½ miles long and is divided into eight sections from 3500 ft. to 4200 ft. in length separated by expansion joints originally 4 in. wide. In four of the sections the slab is 7 in. thick, and 8 in. in the others. The amount of reinforcement is from 0·3 per cent. to 0·5 per cent., and from 0·7 per cent. to 1 per cent. Strain gauges were effective for the first year only. The maximum stress observed across a construction joint was 63,400 lb. per square inch, which is much less than the yield-point stress of the bars. The stress away from the joints did not exceed 10,000 lb. per square inch, indicating that the concrete was resisting a large part of the tensile forces. The spacing of the cracks is related to the amount of reinforcement, and the width of the cracks is least in the sections with 1 per cent. of reinforcement and widest in the sections with 0·3 per cent. The cracks are more frequent in the zones 300 ft. to 500 ft. from the ends. Most cracks are too narrow to be easily seen. Although it is too early to draw definite conclusions, it is thought that a continuously-reinforced concrete road will give excellent performance, but that it may be necessary to provide more reinforcement than earlier studies indicated.

Two test roads were laid in New Jersey in the autumn of 1947, one 5430 ft. long and 8 in. thick containing 0·9 per cent. of reinforcement, and the other 5130 ft. long, 10 in. thick, and with 0·72 per cent. of reinforcement. Both roads have two independent 12-ft. strips. The reinforcement is double cold-drawn welded wire fabric in which the longitudinal wires are ⅜ in. diameter at 3 in. centres. The 8-in. slabs are laid on 12 in. to 14 in. of material of good quality and the 10-in. slabs on 12 in. of the same material. The soil is highly susceptible to pumping. Very heavy vehicles are common on this road. Cracking, except at the ends, is very extensive and erratic, and is greater in the 8-in. slab and more numerous in the outer strips which are subjected to the greatest traffic. Spacing of the cracks is from 6 in. to 20 ft., although it seldom exceeds 12 ft. There is some spalling at the cracks, and where they are close together there is a tendency to form separate small pieces of concrete which shatter and become dislodged. Serious cracks occurred in the newer concrete at construction joints; this suggests the advisability of tying together slabs of different ages, especially if there is a delay of several days between casting the two slabs. The cracks are much wider than they should be if long service is required, and there is ravelling at the cracks which, if it continues, will be serious. The 8-in. slab with the larger percentage of reinforcement appears to be in better condition than the 10-in. slab.

A Canal Cover and Culvert at Cardiff.

A NEW dual-carriageway road, called Churchill Way, Cardiff, was opened in 1949 and is constructed in part over a non-navigational canal conveying water to the docks. The length of the covered canal is about 1700 ft., and the initial design was for a reinforced concrete twin box-culvert. Because of the scarcity of timber and steel and the difficulty of maintaining the flow of water during construction, the design adopted is a single rectangular culvert comprising walls of plain concrete 20 ft. apart and a covering of precast reinforced concrete beams (*Fig.* 1). When possible the water level in the canal was lowered to enable work to be done, but it was not practicable to provide a cast-in-situ concrete beam-and-slab cover because for the most of the length there is insufficient distance between the ordinary level of the water and the underside of the cover to allow the use of shuttering. The cover was designed to carry the Ministry of Transport loading for trolley-bus routes.

The cofferdams within which the walls were constructed consisted of steel sheet-piles driven along the centre-line of the canal for a length of about 20 ft., the ends of the piling returning to one bank. Excavation and concreting of this length of one wall and regrading of part of the bed of the canal were done in the dry, after which the ends of the cofferdam were changed to the other bank and the corresponding length of wall opposite constructed.

Fig. 1.—Canal Cover at Cardiff.

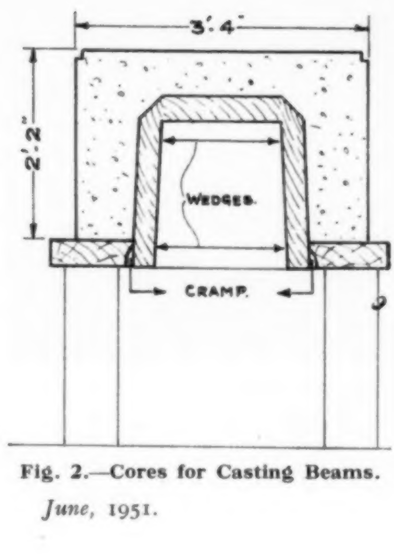


Fig. 2.—Cores for Casting Beams.

The beams are of inverted-channel section (*Fig.* 2), 24 ft. long, 3 ft. 4 in. wide, and 2 ft. 2 in. deep. The vertical ribs are 8 in. wide and the top slab is 6 in. thick. The ends are solid for a length of 2 ft. to provide bearing surfaces on the walls, and the ribs are stiffened at two points by 6-in. diaphragms. The moulds were designed so that the cores could be extracted before the beams were fully matured so that the cores could be used to cast 21 beams each week. The wooden cores were lined with sheet metal to prevent the timber swelling during concreting, and a comparatively large " draw " was provided. The moulds were erected on elevated platforms so that after 24 hours the cores could be extracted from below without disturbing the main

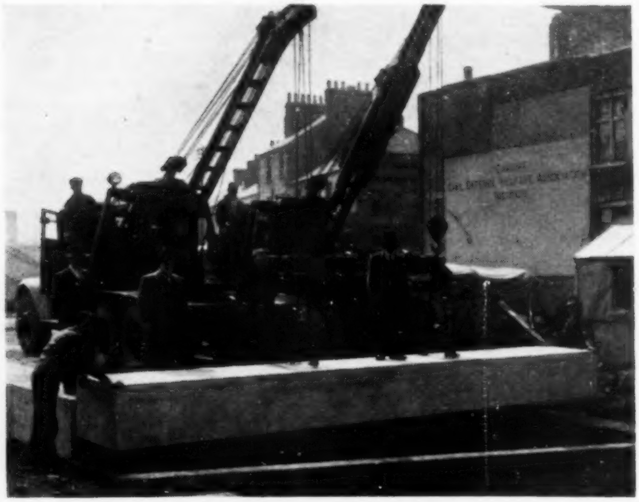


Fig. 3.—Placing Beams in Position.

Fig. 4.—Top Slab of Cast-in-Situ Culvert.

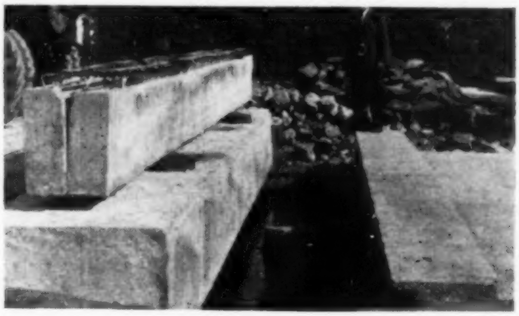


Fig. 5.—Closure Beams.

runners on which the beams rested. The beams remained in position for ten days before being lifted. Rapid-hardening Portland cement was used. The method of extracting the cores was to expand them initially by wedges. After casting, the wedges were removed and a cramp (*Fig.* 2) applied to the outer lower edges. The runners were notched at intervals to provide a space for the ends of the cramps, which drew the sides of the cores away from the concrete. The reinforcement was assembled before being placed in the mould. The concrete was compacted by two electrically-operated vibrators attached to opposite sides of the moulds.

The beams were placed in position by two mobile cranes (*Fig.* 3). The beams weigh $7\frac{1}{2}$ tons each, and were brought from the casting works on special lorries immediately before erection to avoid double handling and stacking at the site.

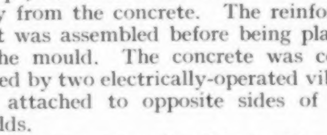


Fig. 6.—Construction Joints.

Two pairs of lifting-links were embedded at the two third-points. The lorry was taken under the two cranes, which lifted the beam, travelled forward, and placed the beam on to the walls on the tops of which cement grout had been placed. As erection proceeded, the cranes travelled over the beams already laid.

It was also necessary to construct an inverted syphon, which is a twin reinforced concrete box-culvert each compartment of which is 10 ft. by 4 ft. Steel sheet-piles were driven in mid-stream to enable the first part of the culvert to be completed and the water was diverted into this while the second part was constructed. The shuttering of the inner faces of the walls of the culvert was ordinary steel panels, but steel sheets, originally the tops of table-shelters, were used for the soffit of the top slab (*Fig.* 4). Each part of the culvert was constructed in three sections, respectively 61 ft. 6 in., 51 ft., and 28 ft. 6 in. long. The middle section is level and lower than the outer sections and the soffit is 21 in. below the original water level. The foundation was covered with rough concrete to receive the reinforced concrete bottom slab, which was cast monolithically with the splays at the bottom of the walls, where keyed longitudinal construction joints were formed (*Fig.* 6). The walls were shuttered and cast up to the underside of the upper splays, where similar construction joints were formed. The final operation was the shuttering and casting of the top slab and splays. The space between the end of the cast-in-situ culvert and the precast beams was closed by a special beam (*Fig.* 5) consisting of two separate

ribs, cast in the same moulds as the ordinary beams but without a top slab. The ribs were placed in position on either side of the gap, which was about 2 in. out of parallel owing to creep in laying the beams due to a slight warping of the moulds. Rebates on the inner faces of the ribs provided a support for asbestos-cement sheets, which were cut to the slight taper required and formed shuttering for a cast-in-situ slab.

The foregoing notes and illustrations are from a paper by the City Surveyor, Mr. E. C. Roberts, M.Eng., M.Inst.C.E., under whose supervision the works were designed and constructed, published in the Journal of the Institution of Municipal Engineers. The main contractors were Messrs. Maberly Parker, Ltd. The beams were made by the British Fram Construction Co. (1911), Ltd., and were placed by Messrs. Robert Wynn & Son, Ltd.

Test of a Prestressed Concrete Railway Bridge.

An under-line prestressed concrete railway bridge (*Fig.* 1) at the Normanby Park Steelworks of Messrs. John Lysaght, Ltd., was recently tested by a locomotive drawing a wagon loaded with steel billets. The structure is a skew bridge, 14 ft. 4 in. wide between the parapets, the angle between the face of the abutments and the longitudinal centre-line of the bridge being 27 deg. 50 min., and the clear span measured at right-angles to the abutments 20 ft. The deck comprises fourteen prestressed precast concrete beams (*Fig.* 2), the length of each of which, because of the skew, is 48 ft. Concrete was cast in situ between and over the beams. The bearing of the beams on the abutments is 2 ft. measured at right-angles to the face of the abutments. The beams are tied together by ⅜-in. twisted square bars passing transversely across the bridge at 2-ft. centres through holes in the webs of the beams. The thickness of the concrete over the beams is 6 in. at mid-span tapering to 3¾ in. at the abutments. The slope of the top of the slab, which is covered with asphalt, facilitates drainage.

The beams are an inverted tee in cross section, 27 in. deep and 8 in. wide; the width of the flange is 13½ in. At the bottom, there are sixty-four 0·2-in. diameter wires having a tensile strength of 100 tons per square inch; there are six-

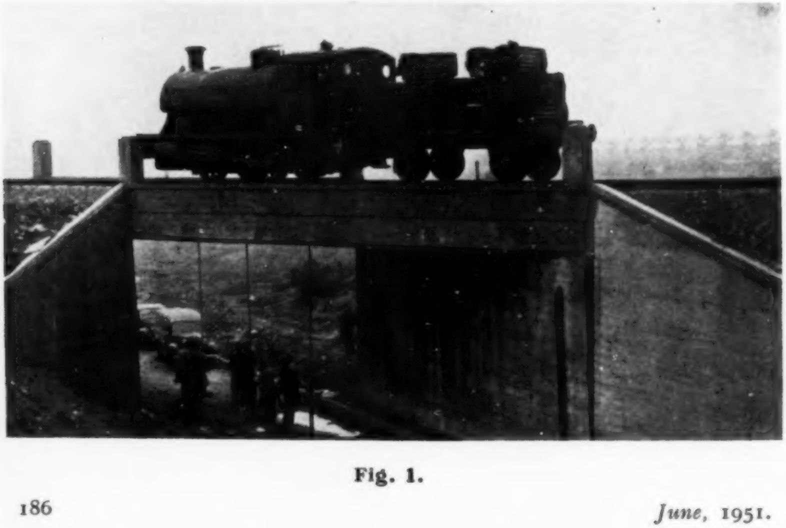


Fig. 1.

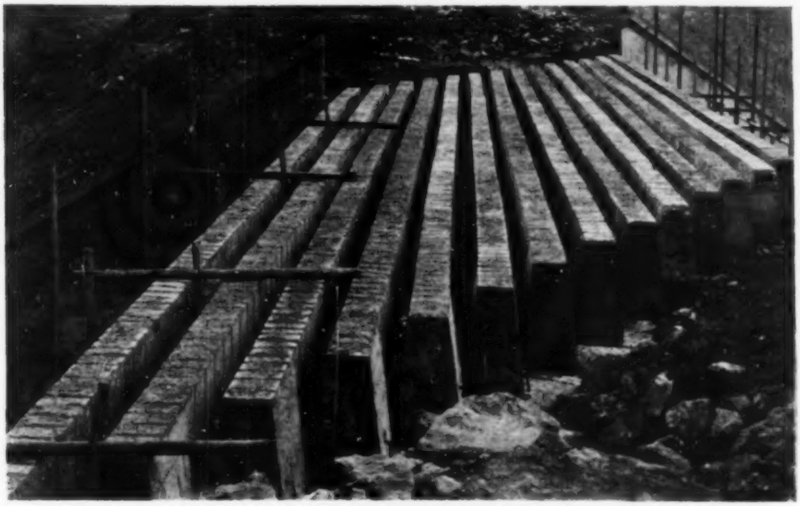


Fig. 2.

teen wires at the top. It is assumed that the precast and cast-in-situ parts of the bridge act together as a monolithic structural member. A description of a test on one of the beams, which confirmed this assumption, was given in this journal for August, 1950.

The weights of the test load were about 37 tons from the locomotive, 32·2 tons from the front axle of the wagon, and 30·2 tons from the rear axle. The train travelled over the bridge three times with the wagon leading, and the deflection was measured under each parapet and at the centre. The maximum deflections were 0·019 in. under the parapet and 0·02 in. at the centre. The ratio of deflection to the span was about $\frac{1}{27,000}$ for the skew-span, or about $\frac{1}{12,000}$ for the right-span.

The bridge was designed by Twisteel Reinforcement, Ltd., and the beams were made by Costain Concrete Co., Ltd. The bridge was built by the Demolition & Construction Co., Ltd.

Conditions of Contract.

THE third edition of "General Conditions of Contract" for civil engineering works was published in March, 1951, by the Institution of Civil Engineers, the Association of Consulting Engineers, and the Federation of Civil Engineering Contractors. The revisions made to the previous edition (January, 1950) include the following. Disputed claims by the contractor are submitted to an arbitrator at the option of the contractor, and in connection with the settlement of disputes the Arbitration Act of 1950 is quoted. Slight alterations have been made to the definition of "excepted risks" and to the provisions to apply upon outbreak of war. The contractor has the exclusive use of plant and material, which when brought on the site by him become the employer's property. A new clause regarding the reinstatement of highways is included. A certificate of completion of parts of a works used by the employer will be awarded upon application by the contractor provided that, unless stated to the contrary, the certificate is not deemed to apply to ground or surfaces needing reinstatement.

A Pipe Bridge in Italy.

THE design and construction of an unusual pressure-main bridge across the river Velino, Italy, is described by G. Turazza in "Beton- und Stahlbetonbau" for October, 1950. The pipe is in the form of an arch of 197 ft. span (*Figs.* 1 and 2) having a radius of 175 ft. and a rise of 30 ft. The internal radius of the circular pipe is 3·28 ft.; the thickness is 10 in. at the crown of the arch increasing to 20 in. at the springings, where the curved pipe joins the horizontal main.

The design of the arch took into account the weight of the pipe and the water, the internal water pressure, shrinking of the concrete, and change of temperature. The stresses due to dead weight were calculated in the ordinary way. In considering the effect of the water in the pipe it was considered that in filling the pipe the water would rise gradually in one half until it flowed over the invert at the crown and, when completely filled, the hydrostatic pressure might be due to a head of not less than the diameter of the pipe or to a maximum head of 57·5 ft. In the case of the pipe being full, the resultant of the hydrostatic pressure was computed by considering a radial sector at O_1 (*Fig.* 3) at an angle ϕ. The y-ordinate of O_1 is $R(1 - \cos\phi)$, and the hydrostatic pressure p_{01} at O_1 is $w(y + h)$, where w is the weight per unit volume of water. The pressure p at point B on the perimeter of the pipe is $p_{01} - w(B'O_1)\cos\phi$, and since $B'O_1$ is $r\cos\psi$ (see the cross section in *Fig.* 3),

$$p = w[h + R(1 - \cos\phi) - r\cos\psi\cos\phi].$$

The corresponding pressure parallel to the vertical diametrical axis of the pipe is $p\cos\psi$ per unit length of pipe and acts on an area da of $\frac{R + (B'O_1)}{R} r.d\psi$,

that is $da = \frac{r}{R}(R + r\cos\psi)\,d\psi$.

Therefore the resultant of all forces acting on unit length of pipe is given by

$$q = 2\int_0^{\pi} p\cos\psi\,da$$

$$= \pi w r^2\left(\frac{h}{R} + 1 - 2\cos\phi\right).$$

Substituting $\pi w r^2 = W$ and $\frac{h}{R} + 1 = k$,

$$q = W(k - 2\cos\phi).$$

Fig. 1.

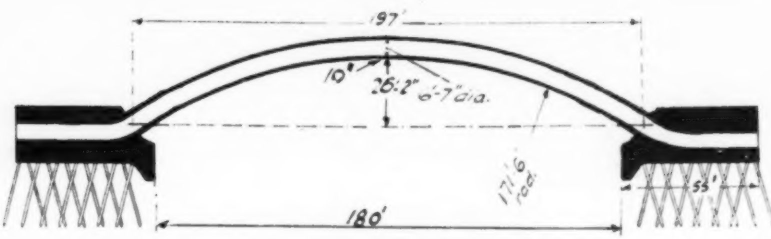


Fig. 2.

VIBRO PILES

at Carrington Power Station

At this site more than 3,000 Vibro piles were driven, the pile lengths ranging from 26 to 50 ft. All pile tops were finished to the required level, a matter of no difficulty with Vibro cast-in-place piles.

Consulting Engineers : C. S. Allott & Son (Manchester).
General Civil Engineering Contractors : A. Monk & Co. Ltd.
Sub-contractors for Vibro piling : John Gill Contractors Ltd.

Photograph by courtesy of British Electricity Authority.

BSP

BRITISH STEEL PILING CO. LTD.

Kings House, 10 Haymarket, S.W.1. Tel. : Trafalgar 1024/8 Grams. : Pilingdom, Lesquare, London

The force q acts radially and is positive when k exceeds $2 \cos \phi$. Since at the crown $\cos \phi = 1$, q is always positive if h exceeds R. When h is less than R, q is negative at some sections near the crown. In this structure, and generally for low-pressure mains, q is always negative and acts as a variable radial force. In high-pressure mains q may be great enough to reverse the shearing force at the crown and produce a tensile force.

In the design of the arch, the resultant of the forces $q.ds$ (*Fig.* 3) between the crown and any section O_1 was calculated, and its components X and Y and moment M about the centre of any section determined. Considering a segmental element (angle $d\alpha$), the force $q.ds$ is $qR.d\alpha$; the component dx is $Rq \sin \alpha.d\alpha$ and dy is $-Rq \cos \alpha.d\alpha$. Hence

$$X = \int_0^\phi dx = RW[k(1 - \cos \phi) - \sin^2 \phi].$$

$$Y = \int_0^\phi dy = RW(\phi + \sin \phi \cos \phi - k \sin \phi).$$

The moment dM of $q.ds$ about O_1 is $qR.d\alpha(O_1F)$; therefore

$$M = \int_0^\phi dM = R^2W[k(1 - \cos \phi) - \phi \sin \phi],$$

which is positive for rotation clockwise.

Values of X, Y, and M at ten equidistant sections were calculated from these formulæ with $h = 3{\cdot}28$ ft. ($k = 1{\cdot}019$). The analysis of the forces in a partly-filled pipe is more complex than for a full pipe. (The derivation of formulæ for and values of q for the pipe with the water at the level of the invert at the crown are given in the original article.) The calculation of the arch was made in accordance with ordinary methods, which gave the following maximum values.

	At Crown.	At Springings.
Thrust (lb.)	− 560,000	− 478,000
Bending moment (ft.-lb.)	+ 2,605,000	− 7,000,000
Compressive stress in concrete (lb. per square inch)	1,081	880
Tensile stress in reinforcement (lb. per square inch)	16,200	15,100

The maximum stresses at the crown are produced by the dead weight, the pipe full of water ($h = 3{\cdot}28$ ft.), and a fall of temperature of 25 deg. C., and at the springings by the dead weight, half the pipe full, and a fall of temperature of 25 deg. C. The maximum tensile stress in the concrete, which occurs when h is 57·5 ft., is due to a tensile force of 51,900 lb. and, with an assumed modular ratio of 40, is 93 lb. per square inch in a pipe 10 in. thick. The maximum compressive stress in the longitudinal direction of the pipe (with $m = 10$) is 853 lb. per square inch, and the maximum compressive stress radially is 17 lb. per square inch. Assuming Poisson's ratio to be 0·125, the principal stress is 202 lb. per square inch. As the pipe had to be watertight it was necessary to avoid cracking, and the two abutments were designed to prevent unequal settlement. Each abutment is supported on 68 piles inclined at 15 deg. from the vertical.

The centering for the arch comprised a cradle carried on three trusses supported on sand boxes on the abutments and on two intermediate temporary piers. The sections against the abutments were concreted completely, the invert of the pipe was then concreted in nine sections working from three intermediate points, and concrete was placed later in the four spaces between the five principal sections.

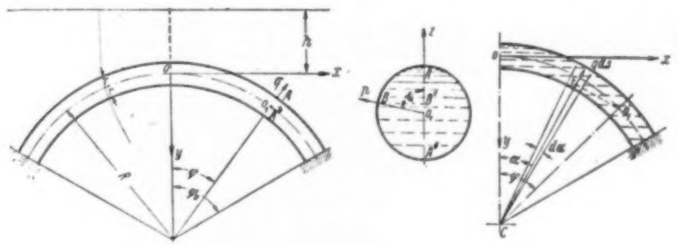
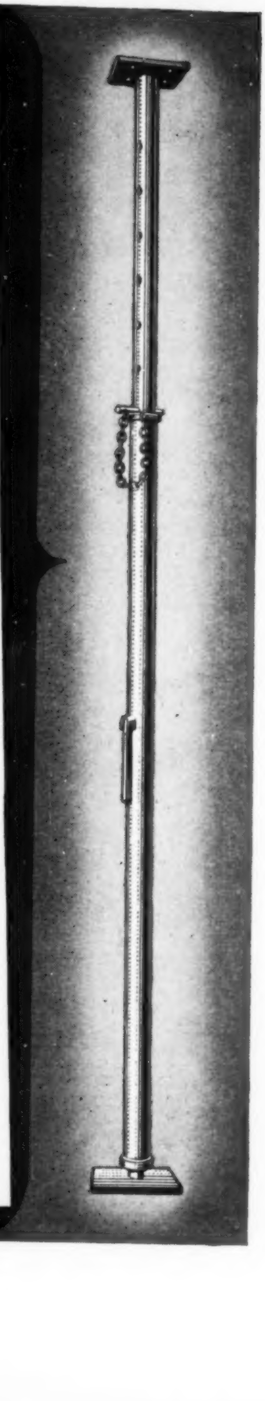


Fig. 3.

Patent Relating to Concrete.

Concrete Improver.

CONCRETE is hardened by applying agents which react with one another to fill the pores, the different agents being mixed separately into an aqueous emulsion of soap and an oil, and the emulsion broken by means of an acid to produce a viscous and oleaginous paste. The material is prepared by dissolving soap (preferably soft soap) in water, stirring in a heavy oil such as colza oil, and then introducing separately ground lime or marble, potassium silicate, and calcium chloride. After thorough mixing and, if necessary, the addition of more water, approximately 3 per cent. by weight of the cement of concentrated hydrochloric acid is added, preferably in the gauging water. If the setting-time is reduced too much, potassium chromate may be included—No. 606,153. H. Avellan. January 7, 1946. [Note. Publication of specifications by the Patent Office has been delayed by the war.]



Exhibition of Prestressed Concrete.

At the Exhibition of Industrial Power at the Kelvin Hall, Glasgow, there are shown prestressed concrete structural members and models showing the methods of manufacture of prestressed concrete and the plant used by various firms who specialise in prestressed concrete. The exhibition has been organised in connection with the Festival of Britain, and remains open until August 18.

Prestressed Concrete Bridges in the U.S.A.

THERE are at present at least three prestressed concrete bridges either complete or in course of construction in the United States. The first to be put into service is a road bridge in Tennessee. The first to be commenced is the Walnut Lane Bridge, Philadelphia, having a span of 155 ft., the design, construction, and testing of which is described fully by Professor Magnel in his book "Prestressed Concrete". The third is a footbridge in California which, together with the bridge in Tennessee, is described in the following abstract from recent numbers of "Engineering News-Record".

Road Bridge in Tennessee.

This bridge has two simply-supported end spans of 20 ft. and a central span of 30 ft. The road is 19 ft. wide. The end and intermediate supports are timber piled frames. Each span comprises fifteen 16-in. by 12-in. prestressed beams side by side, and cast-in-situ top slab, kerbs, and filling between the beams. The beams, which are held together by transverse cables, are composed mostly of standard machine-made 8-in. by 16-in. by 12-in. concrete blocks, with special end-blocks in which the longitudinal wires are anchored, and depresser blocks which hold the wires in place vertically. The standard blocks have three rectangular openings through which the wires pass, and a bottom flange that projects about 1 in. beyond the sides. The blocks and the mortar between them have a compressive strength of 3750 lb. per square inch.

The beams, of which there are 45, were made in a factory. One beam was assembled at a time on a table, starting with an end-block with the wires attached. One wire is provided in each 20-ft. beam and two wires in each 30-ft. beam. The wires are composed of seven strands each, galvanised for protection against corrosion, and are $\frac{5}{16}$ in. diameter. A fitting is attached to each end and a threaded stud is inserted in the fitting. Turning a nut on the stud adjusts the tension. The nut bears against a 5-in. by 6-in. steel plate $\frac{3}{4}$ in. thick, which distributes the pressure over the end-block. The blocks, the ends of which were mortared, were strung on the wires and pressed together. The other end-block was then attached and an initial tensile force of 10,000 lb. per wire applied; this is equivalent to an average compressive stress in the concrete of about 100 lb. per square inch for single-wire beams. The next day

the final force of 26,000 lb. per wire was applied by an hydraulic jack, and is equivalent to about 125,000 lb. per square inch in the wires, that is, less than the stress at which creep occurs. In course of time the tensile stress may decrease to 105,000 lb. per square inch, producing an average compressive stress in the concrete in a single-wire beam of about 230 lb. per square inch. At mid-span the compressive stress at the bottom of a prestressed single-wire beam is about 470 lb. per square inch while carrying its own weight, and in a double-wire beam up to 940 lb. per square inch. There is no stress in the top of the beams under this condition.

The beams were put in place by a crane, those for one end span being placed first. Wooden planks 3 in. thick were placed on these beams and the crane travelled on them to erect the beams for the middle span. The crane then moved on the middle span to place the beams for the other end span. As the beams were placed, seven-strand wires for the transverse prestressing were threaded through holes in the blocks. When the cast-in-situ slab had been placed and had hardened, each of the transverse wires was tensioned to 26,000 lb. with a jack.

The beams were assembled and prestressed in three days, and the slab and kerbs were cast in one day. Rapid-hardening Portland cement was used, and the bridge was put into service two weeks after erection started.

Footbridge near Los Angeles.

A prestressed concrete footbridge of 110-ft. span is in course of construction near Los Angeles. The 5-in. deck slab is 8 ft. wide and is designed for a live load of 55 lb. per square foot. The two simply-supported girders, projecting 4 ft. above the slab, are 5 ft. 8 in. deep, the webs being 10 in. thick and the top flange 20 in. wide and 9 in. deep. Each girder is prestressed with 136 wires of 0·2 in. diameter. The initial stress in the wires, which are stretched five at a time, is

120,000 lb. per square inch. The working compressive stress in the concrete is 1700 lb. per sq. in. and the final tensile stress in the wires is 102,000 lb. per square inch. The method of stretching the wires is as follows.

Five wires are threaded through holes in a 1-in. by 4-in. steel block 3½ in. thick, and ⅝-in. washers are slipped over them. A button-head is attached to the end of each wire. This fastening is expected to yield not more than ⅛ in. when the force in the wire is 1½ times the force at the working load. The group of wires is then inserted in a metal sheath to prevent bond with the concrete, and placed in the shuttering. The concrete is then placed and, when it attains a strength of 5000 lb. per square foot (in about 28 days), jacks bearing against a 6½-in. by 20-in. steel block 3 in. thick are used to stretch the wires. When all the groups of wires are stretched, the space around them is grouted under pressure through 1-in. holes in the jacking blocks. The girders, each of which weighs about 50 tons, were precast at the site, a method which reduced the cost of the shuttering compared with cast-in-situ construction, and enabled the girders to be made during the winter when falsework in the river was not practicable. Each girder contains 50 cu. yd. of concrete and 7000 lb. of steel, compared with 88 cu. yd. of concrete and 40,000 lb. of steel required for an ordinary reinforced concrete bridge.

(*Continued on facing page.*)

British Standard for Concrete Mixers.

British Standard No. 1305 (1946), "Batch Type Concrete Mixers", now includes details of a 2 cu. ft. tilting mixer. The ratio of the volume of the drum to the nominal capacity must be not less than 2·53 nor more than 3·18. A tank for measuring water is not necessary if a power-loader is not fitted, but otherwise the tank must have a capacity of at least 2½ gallons and be capable of measuring and discharging automatically ½ gallon of water. The minimum height above the ground of the discharge point of a portable 2 cu. ft. mixer is 2 ft.

(Continued from facing page.)

SITUATIONS VACANT. The Docks and Inland Waterways Executive invite applications for the following temporary positions in the Office of the Chief Engineer for Docks, Hull: One senior reinforced concrete designer; Two reinforced concrete detailers; Two civil engineering draughtsmen. The salary for the positions will be from £550/£680 per annum but additional allowances above the scale may be given for suitably qualified and experienced applicants. The work in the first instance will be the design and preparation of contract drawings, specification, and bill of quantities for a deep water tidal wharf of reinforced concrete to carry travelling cranes and railway tracks. The senior designer should have initiative and be able to take charge with a minimum of supervision; he should preferably have a knowledge of prestressed concrete. Experience of associated harbour and dock work would be an advantage for all the appointments. Whilst the positions will be temporary, there will be a prospect of permanency. Applications stating age, qualifications, experience, present position, and salary, etc., should be sent to reach the Secretary, 22 Dorset Square, London, N.W.1, not later than 25 June, 1951.

SITUATION VACANT. Well-known London firm specialising in hollow-tile floor construction require an assistant (to their chief engineer) to take charge of and control drawing-office. Applicants must have knowledge of reinforced concrete frames and steelwork and various forms of construction, be familiar with drawing-office practice and site work, and possess experience in the preparation of designs, calculations, and detailing. Write fully experience, technical qualifications, and salary required. Box 2478, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

WANTED. Architectural draughtsman. Good 'bus and rail service. Canteen facilities. Applicants should state age, experience, and salary required in confidence to The Secretary, The Croft Granite, Brick and Concrete Co., Ltd., Croft, near Leicester.

SITUATIONS WANTED.

SITUATION WANTED. B.Sc., A.M.I.Struct.E., 36, going to seed, seeks change. Experience in reinforced concrete frames, reservoirs, dams, sewage and water, pumping stations, etc. Starting salary £800. Box 2473, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATION WANTED. Reinforced concrete designer with experience is able to undertake part-time work at home for consulting engineers. Design and detailing of all types of reinforced concrete structures. Write giving your requirements to Box 2477, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

FOR SALE.

FOR SALE. New 2½-in. bore rubberised canvas suction hose with coiled rust-proofed wire insert in 12 ft. and 6 ft. lengths. 10s. and 5s. per length, carr. paid. Small orders C.W.O. please. Woodfield & Turner, Burnley. Telephone: Burnley 3065.

FOR SALE. Sacks, bags, and curing cloths for sale. You want the best type and quickest delivery. Write John Braydon, Ltd., 26 The Highway, London, E.1. Telephone: ROYal 1044.

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(Delivered in London area.)

Aggregates (per cu. yd.).—Washed sand, 17*s*. Clean shingle: ¾ in., 15*s*. 2*d*.; ⅜ in., 17*s*. 6*d*. Thames ballast, 15*s*. 9*d*. Broken brick, ¾ in., 17*s*. 6*d*.

Cement (per ton, delivered at Charing Cross).—
Portland cement, 6 tons and upwards, 91*s*. 1 ton to 6 tons, 96*s*. Paper bags and non-returnable jute sacks included.
Rapid-hardening Portland, 6*s*. above ordinary Portland.
Aquacrete and 417, 32*s*. 6*d*. above ordinary Portland; paper bags included.
Colorcrete (buff, red, and khaki), in 6-ton loads, 132*s*. 6*d*.; paper bags included.
Snowcrete, £12 8*s*. 6*d*., inc. paper bags.
"Super-Cement," 31*s*. 6*d*. per ton above ordinary Portland cement; bags included.
High-alumina cement, 1 ton and upwards, 247*s*. 6*d*. per ton; paper bags 17*s*. 6*d*. per ton extra.
Snowcem paint, 56*s*. per cwt. inc. containers.

Shuttering.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1*s*. 1*d*.) and No. 94 (price 5*d*.) issued by H.M. Stationery Office.

Reinforcement.—Mild steel round bars (per cwt.): ⅝ in. to 2⅞ in., 28*s*. 2*d*. 7/16 in. to ½ in., 29*s*. ⅜ in., 29*s*. 8*d*. ¼ in., 31*s*.

Materials and Labour.

(Contracts up to £5000. Inc. 10 per cent. profit.)

Portland Cement Concrete, 1 : 2 : 4.—
Foundations, 2*s*. 2*d*. per cu. ft. Columns, 2*s*. 5*d*. per cu. ft. Beams, 2*s*. 5*d*. per cu. ft. Floor slabs 4 in. thick, 6*s*. 10*d*. per sq. yd.; Do., 5 in., 8*s*. 7*d*.; Do., 6 in., 10*s*. 3*d*.; Do., 7 in., 12*s*. Walls 6 in. thick, 10*s*. 3*d*. per sq. yd. Add for hoisting 3*s*. 6*d*. per cu. yd. above ground floor level. Add for rapid-hardening Portland cement 2*s*. per cu. yd.

Reinforcement.—Mild steel round bars, including cutting, bending, fixing, and wire (per cwt.)—¼ in. to ⅜ in., 48*s*. 6*d*. 7/16 in. to ½ in., 43*s*. 6*d*. 9/16 in. to 2⅞ in., 42*s*.

Shuttering and Supports.—
Walls, 155*s*. per square.
Floors (average 10 ft. high), 160*s*. per square. In small quantities, 1*s*. 8*d*. per sq. ft.
Columns, average 18 in. by 18 in. (per sq. ft.), 1*s*. 8*d*.; in narrow widths, 2*s*.
Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2*s*.; in narrow widths, 2*s*. 2*d*.
Raking, cutting, and waste, 5*d*. per lin. ft.
Labour on splays, 3*d*. per lin. ft.
Small fillets to form chamfers, 6*d*. per lin. ft.

Wages.

The rates of wages on which the above prices are based are: Carpenters and joiners, 3*s*. per hour (carpenters 2*d*. a day tool money); Labourers, 2*s*. 6*d*.; Men on mixers and hoists, 2*s*. 7*d*.; Bar-benders, 2*s*. 8*d*.

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